Chapter 2
Cast-in-Place Conduits for Dams

2-1. General

The selection of the most economical conduit cross section must depend on the designer's judgment and the consideration of all design factors and site conditions for each application. For fills of moderate height, circular or rectangular openings will frequently be the most practicable because of the speed and economy obtainable in design and construction. For openings of less than about 5.6 m^2 (60 ft^2), a single rectangular box probably will be most economical for moderate fills up to about 18.3 m (60 ft). However, a rectangular conduit entrenched in rock to the top of the conduit may be economical for higher fills since the applied vertical load need be only the weight of the earth directly above with no increase for differential fill settlement. The ratio of height to width should be about 1.50 to accommodate the range of loading conditions economically. Where there is a battery of outlet gates, a multiple-box shape is sometimes economical where acceptable from a hydraulic standpoint.

a. Single conduits. For a single conduit of more than about 5.6-m^2 (60-ft^2) area and with a fill height over 18.3 m (60 ft), it will generally be found economical to use a section other than rectangular for the embankment loading (Condition III). The circular shapes are more adaptable to changes in loadings and stresses that may be caused by unequal fill or foundation settlement. For cases in which the projection loading condition applies, no material stress reduction results from the provision of a variable cross section. These structures should be formed as shown in Figure 2-1 and should be analyzed as a ring of uniform thickness. While these sections show variations in thickness in the lower half of the conduit due to forming and other construction expedients, such variations may be disregarded in the design without appreciable error.

b. Oblong sections. The oblong section shown in Figure 2-1 is formed by separating two semicircular sections by short straight vertical wall sections. The oblong section generally achieves maximum economy of materials by mobilizing more of the relieving fill pressure. The proportions should be selected carefully, and the tangent-length-to-radius ratio will usually be between 0.5 and 1.0. The conduit design should cover a range of possible loading conditions, from initial or construction condition to the long-time condition. Here also, a geologist

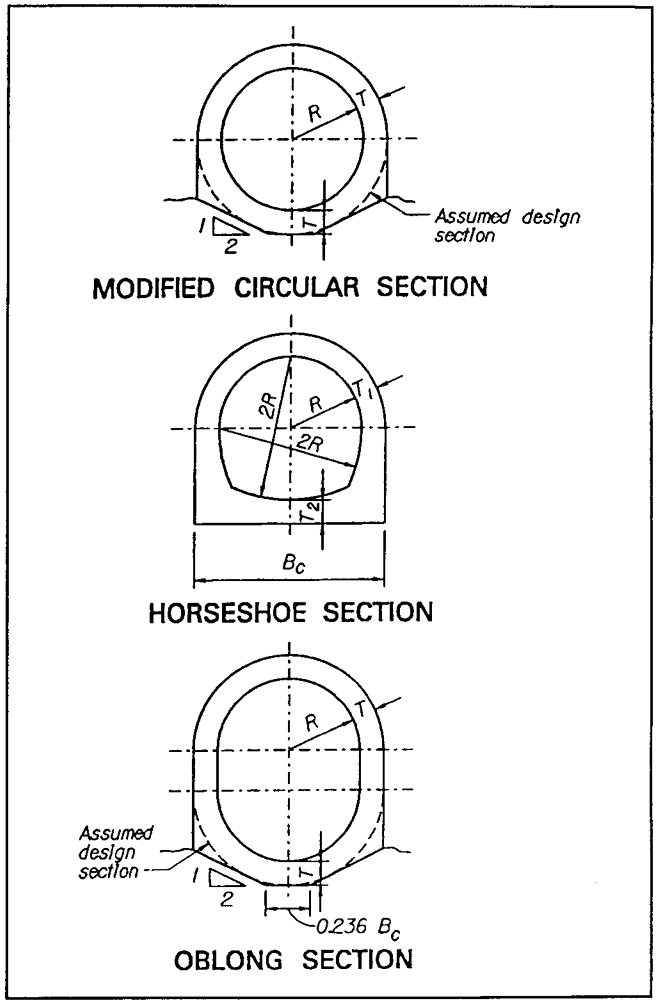


Figure 2-1. Typical cast-in-place conduits

or soils engineer should be consulted before final determination of the base shape of a conduit.

c. Horseshoe sections. The "horseshoe" section in Figure 2-1 is generally less economical than the oblong and is therefore not often used. Its stress distribution is not as desirable as that of the circular or oblong section, and shear stirrups may be required in the base. It may be practicable, however, for some foundation conditions where the fill height is low.

d. Interbedded foundations. It may be difficult to shape the foundation excavation when in closely bedded, flat-lying shale, or when in rock with frequent shale interbeds. For this condition, it may be economical to excavate the foundation level and backfill to the desired shape with a low-cement-content concrete. A geotechnical engineer should be consulted to help develop the excavation plan. Excavation drawings should show the pay excavation lines and not the actual excavation lines. For a conduit under a dam, the designer should show the actual excavation lines rather than the pay excavation lines and the contractor should limit excavation to the actual excavation lines.

2-2. Materials

a. Concrete. Minimum compressive strength 28 MPa (4,000 psi) air entrained.

b. Reinforcement. Minimum yield strength, Grade 400 MPa (60,000 psi).

2-3. Installation

Conduits through dams are cast directly against the soil or rock and, therefore, bedding is not a design consideration. When overexcavation of the foundation materials is required, concrete fill should be used to maintain proper conduit grade. All foundation materials for cast-in-place conduits should be reviewed by a geotechnical engineer.

2-4. Loadings

Typical conduit loads are shown in Figure 2-2. The conduit supports the weight of the soil and water above the crown. Internal and external fluid pressures and lateral soil pressures may be assumed as uniform loads along the horizontal axis of the conduit when the fluid head or fill height above the crown is greater than twice the conduit diameter or span. Foundation pressures are assumed to act uniformly across the full width of cast-in-place conduits. Uplift pressures should be calculated as uniform pressure at the base of the conduit when checking flotation.

a. Groundwater and surcharge water. Because of the ratio of vertical to horizontal pressure, the most severe loading condition will generally occur when the reservoir is empty and the soil is in a natural drained condition. However, the following loads occur where there is groundwater and/or surcharge water.

(1) Vertical pressure. Use Equation 2-1 to determine vertical pressure due to the weight of the natural drained soil above the groundwater surface, the weight of the submerged soil below the groundwater surface, and the weight of the projected volume of water above the conduit, including any surcharge water above the fill surface.

$$W_w = \gamma_d H_d$$

or

$$W_w = \gamma_d H_d + \gamma_s H_s \qquad (2\text{-}1)$$

$$W_w = \gamma_w H_w + (\gamma_s - \gamma_w) H_s$$

where

W_w = vertical pressure due to prism of soil above pipe, N/m^2 (psf)

γ = soil unit weight; d = dry, s = saturated, w = water, N/m^3 (pcf)

H = soil height; d = dry, s = saturated soil, m (ft)

H_w = water height above the point of interest, m (ft)

(2) Horizontal pressure. Horizontal pressure from the lateral earth pressure is obtained by using soil weights for the appropriate moisture conditions and full hydrostatic pressure.

b. Internal water pressure. Internal water pressure should be considered but will seldom govern the design for the usual type of outlet works. However, internal pressures must be analyzed as indicated in Equation 2-2 for pressure conduits for interior drainage in local protection projects.

$$W_i = \gamma_w (H_G \pm r) \qquad (2\text{-}2)$$

where

W_i = internal pressure at point of interest, N/m^2 (psf)

γ_w = unit weight of water, 9.8 kN/m^3 (62.4 pcf)

H_G = hydraulic gradient above point of interest, m (ft)

r = inside radius of conduit, m (ft)

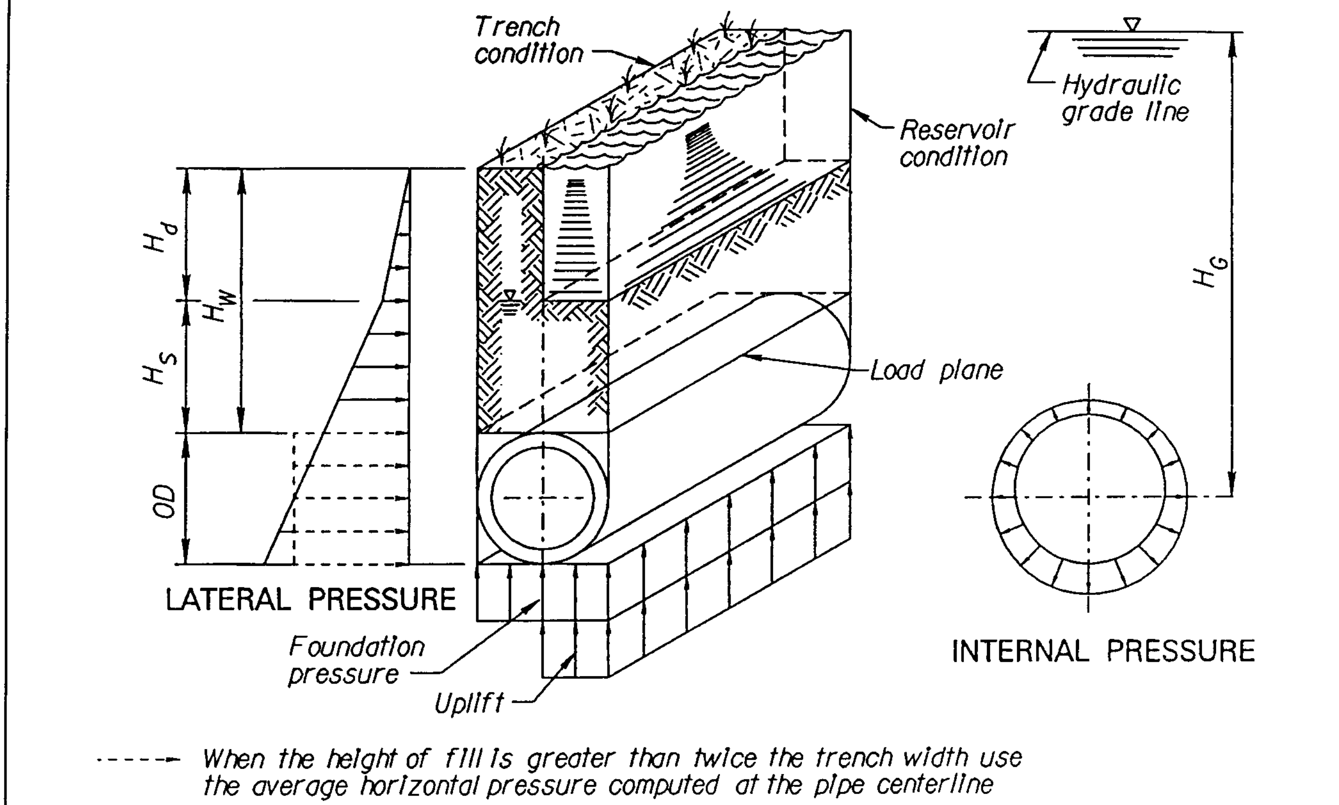


Figure 2-2. Typical conduit loadings

c. Concentrated live loads.

(1) Vertical pressure. Because soil conditions vary, designers can expect only a reasonable approximation when computing vertical pressures resulting from concentrated surface loads. The Boussinesq method is commonly used to convert surface point loads to vertical stress fields through the geometric relationship shown in Equation 2-3. This equation may be used for all types of soil masses including normally consolidated, overconsolidated, anisotropic, and layered soils. Stresses calculated by using this method are in close agreement with measured stress fields, and examples for using Equation 2-3 are shown in Figure 2-3.

$$W_c = \frac{3Pz^3}{2\pi R^5} \tag{2-3}$$

where

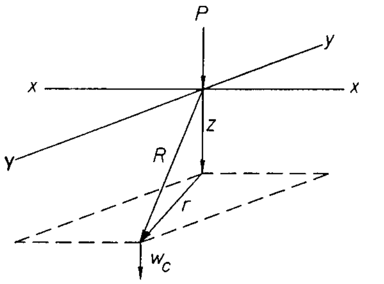
W_c = vertical pressure due to concentrated load, N/m^2 (psf)

P = concentrated load, N (lb)

z = depth to pressure surface, m (ft)

R = radial distance to pressure surface, m (ft)

(2) Horizontal pressure. Lateral loads caused by vehicles can be safely ignored due to their transient nature. However, a minimum lateral pressure of 0.005 of the wheel load for vehicles to a depth of 2.4 m (8 ft) should be considered in accordance with American Society for Testing and Materials (ASTM) C 857. For stationary surcharge loads, a lateral pressure can be calculated by using a Boussinesq equation such as Equation 2-4.



Given: P=44.5 kN (10,000 lbs.)
z=0.92 m (3 ft)
r=0.61 m (2 ft)
$R = \sqrt{r^2+z^2}$
b=0.52 m (1.7 ft) radius of a circular area

L=B=0.92 m (3 ft) dimension of a square area

L=Long side of rectangular area

MINIMUM LATERAL PRESSURES

$p_e = 0.4\, w_c$ (AREA)
$p_e = 0.5$ or $1.0 w_c$ (COE)

METHOD	FORMULA	LOAD PLANE PRESSURE AT POINT Kpa (psf)	CENTER Kpa (psf)
BOUSSINEQ	$w_c = \frac{3Pz^3}{2\pi z R^5}$	10.3 (215)	25.1 (525)
SQUARE AREA	FIGURE 2-5	10.5 (220)	18.4 (384)
CIRCULAR	FIGURE 2-6 $\frac{w_c}{w_o} = [1.0 - \frac{1}{[1+(b/z)^2]^{3/2}}]$	10.5 (220)	18.3 (383) 17.8 (372)
SIMPLIFIED 1 2	$w_c = \frac{P}{(B+z)(L+z)}$ $w_c = \frac{P}{(B+z)^2}$	13.1 (275)	13.1 (275)
AASHTO 1 2 1.75	Pyramid (2 ft) minimum cover	17.2 (359)	17.2 (359)

Figure 2-3. Typical live load stress distribution

$$p_c = \frac{P}{2\pi}\left(\frac{3zr^2}{R^5} - \frac{(1-2\mu)}{R(R+z)}\right) \qquad (2\text{-}4)$$

where

p_c = horizontal pressure from concentrated load, N/m^2 (psf)

r = surface radius from point load P, m (ft)

R = radial distance to point in question, m (ft)

μ = Poisson's ratio, 0.5 for saturated cohesive soils or 0.2 to 0.3 for other soils

Consult a geotechnical engineer for lateral loads from other surcharge conditions.

(3) Wheel loads. For relatively high fills, Equation 2-3 will give reasonably accurate results for highway and railroad wheel loads and the loads on relatively small footings. However, where the conduit is near the surface or where the contact area of the applied load is large, these loads must be divided into units for a more accurate analysis. The use of influence charts as developed by Newmark (1942) will be helpful in computing the stress due to loads on relatively large and irregular areas.

d. Backfill. The behavior of the soil pressures transmitted to a conduit or culvert by the overlying fill material is influenced by the physical characteristics and degree of compaction of the soil above and adjacent to the conduit or culvert as well as the degree of flexibility and the amount of settlement of the conduit or culvert. The effect of submergence in the backfill must also be considered as indicated in Figure 2-2. Direct measurements of such pressures have been made for small-diameter pipes under relatively low fills. Until more data are available, the following loading should be used for rigid conduits and culverts for dams and levees and outlet conduits for interior drainage. The effect of submergence in the backfill must be considered. The three typical conduit installation conditions are trench, trench with superimposed fill, and embankment. Terms for these loading conditions are defined in Figure 2-4.

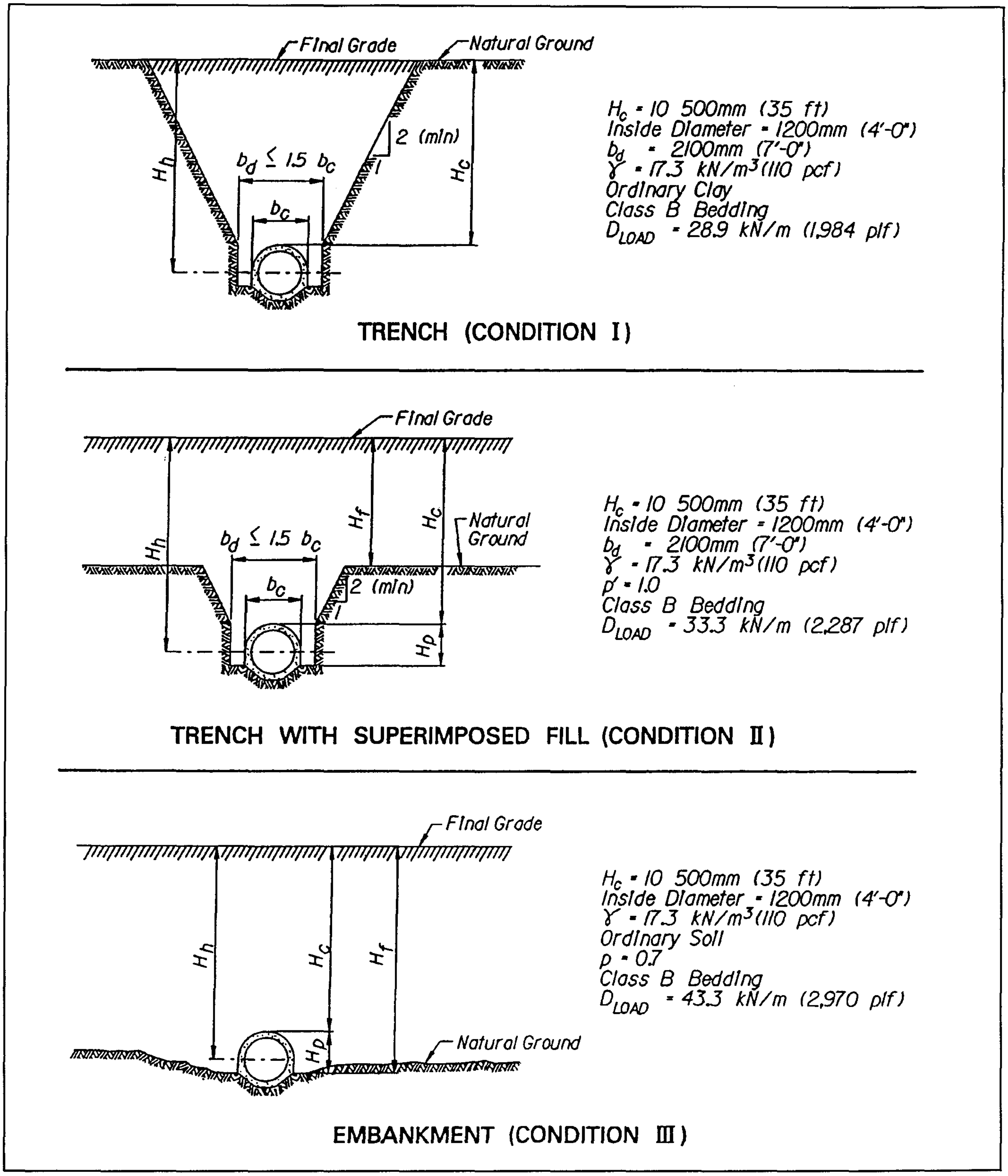


Figure 2-4. Loading conditions for conduits

(1) Trench with no superimposed fill (Condition I).

(a) Loads from the trench backfill condition are applied to those structures that are completely buried in a trench with no superimposed fill above the top of the trench. To satisfy this condition, the width of the trench measured at the top of the conduit should be no greater than one and one-half times the overall width of the conduit, and the sides of the ditch above the top of the conduit should have a slope no flatter than one horizontal to two vertical. The total dead load of the earth at the top of the conduit should be computed as the larger of the two values obtained from Equations 2-5 through 2-7.

$$W_e = C_d \gamma B_d^2 \tag{2-5}$$

or

$$W_e = \gamma B_c H \tag{2-6}$$

$$C_d = \frac{1 - e^{-2K\mu' \frac{H}{B_d}}}{2K\mu'} \tag{2-7}$$

where

W_e = total dead load of earth at top of conduit, N/m (lbf/ft)

C_d = trench coefficient, dimensionless

B_d = trench width at top of conduit < 1.5 b_c, m (ft)

B_c = outside diameter of conduit, m (ft)

H = variable height of fill, m (ft). When $H_c \geq 2B_d$, $H = H_h$. When $H_c < 2B_d$, H varies over the height of the conduit.

μ' = soil constant, dimensionless

Values for $K\mu'$ and C_d can be taken from Figure 2-5.

(b) When the height of the fill above the top of the conduit (H_c) is less than twice the trench width, the horizontal pressure should be assumed to vary over the height of the conduit. When H_c is equal to or greater than 2_{Bd}, the horizontal pressure may be computed at the center of the conduit using an average value of H equal to H_h applied uniformly over the height of the conduit. When $H_c < 2B_d$, the horizontal pressure in N/m^2 (psf) at any depth should be computed using Equation 2-8.

$$p_e = \gamma H \tan^2\left(45 - \frac{\phi}{2}\right) = K_a \gamma H \tag{2-8}$$

where

p_e = horizontal earth pressure, N/m^2 (psf)

γ = unit weight of fill, N/m^3 (pcf)

ϕ = angle of internal friction of the fill material, degrees

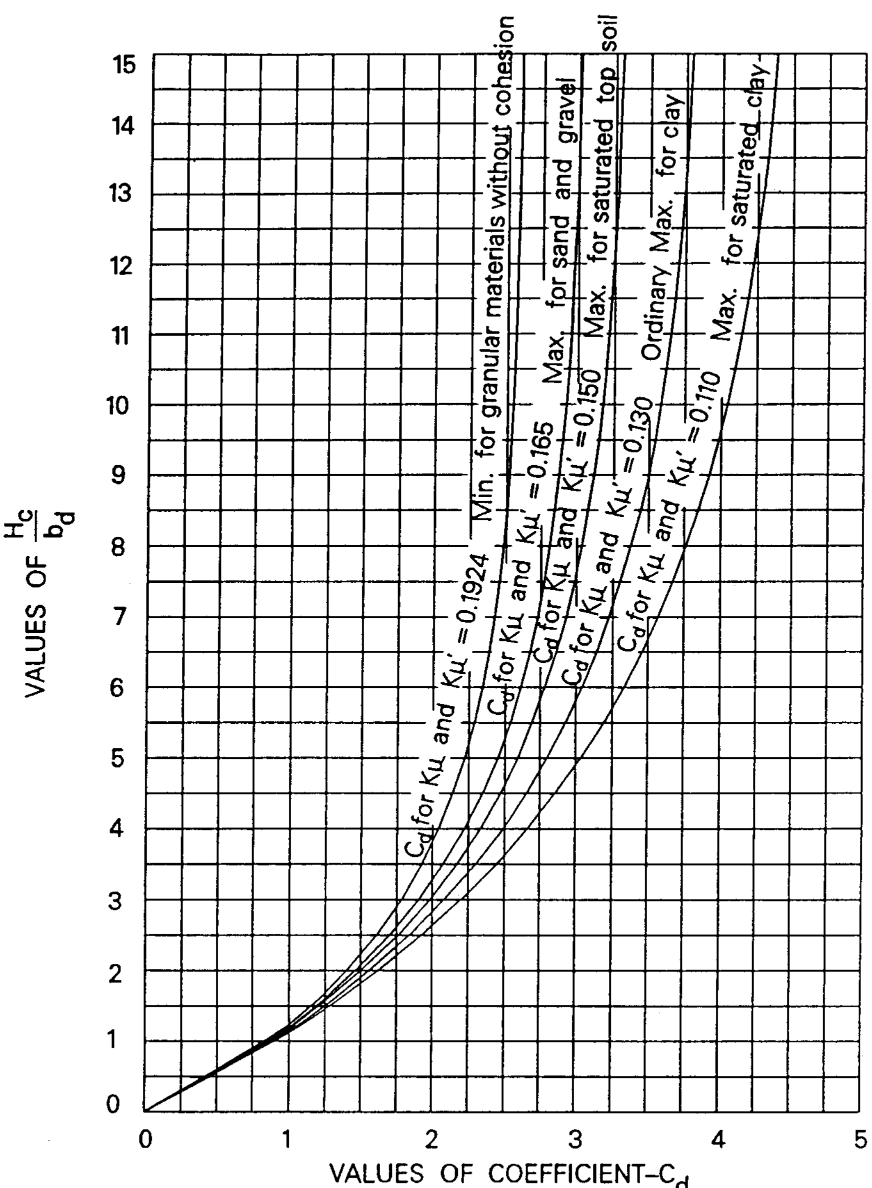
K_a = active pressure coefficient, N (lb)

(c) In most cases, the unit weight and the internal friction angle of the proposed backfill material in dry, natural drained, and submerged conditions should be determined by the laboratory and adapted to the design. However, where economic conditions do not justify the cost of extensive investigations by a soils laboratory, appropriate values of unit weight of the material and its internal friction angle should be determined by consultation with the soils engineer.

(d) Where submergence and water surcharge are applicable, the loadings must be modified. To obtain the total vertical load, the weight of the projected volume of water above the conduit, including any surcharge water above the fill surface, is added to the larger value of W_e obtained by using the submerged weight of the material used in Equations 2-5 and 2-6. The horizontal pressure is obtained by adding the full hydrostatic pressure to the pressure found by Equation 2-8 using the submerged weight of material.

(2) Trench with superimposed fill (Condition II).

(a) This loading condition applies to conduits that are completely buried in a trench with a superimposed fill H_f above the top of the trench. The trench width and side slopes have the same limitations as specified for the trench condition. The vertical and horizontal unit loads for this loading condition vary between the computed values for the Conditions I and III (trench and embankment conditions) in proportion to the ratio $H_f/(H_c + H_p)$. The vertical load, in N/m (pounds per foot) of conduit length, for the Condition II (trench with superimposed fill) should be computed as the larger of the two values obtained from Equations 2-9 and 2-10.



Load per unit of length.
$W_e = C_d \gamma b_d$ [M,KN²(ft. lb.)]
γ = *unit weight of fill materials.*
b_d = *breadth of ditch at top of structure*
H_c = *height of fill over top of structure*

μ = *the "coefficient of internal friction" in the fill materials, abstract number.*
μ' = *the "coefficient of sliding friction" between the fill materials and the sides of the ditch, abstract number.*

$$K = \frac{\sqrt{\mu^2+1} - \mu}{\sqrt{\mu^2+1} + \mu}$$

Figure 2-5. Earth loads trench condition

$$W_e = C_d \gamma b_d^2 + \left(\frac{H_f}{H_c + H_p} \right) (1.5\ \gamma b_c H_h - C_d \gamma b_d^2\ ) \tag{2-9}$$

or

$$W_e = \gamma b_c H_h + \left(\frac{H_f}{H_c + H_p} \right) (1.5\ \gamma b_c H_h - \gamma b_c H_h\ ) \tag{2-10}$$

where

γ = unit weight of fill, N/m^3 (pcf)

b_d = trench width, m (ft), $b_d \leq = 1.5\ b_c$

H_f = height of superimposed fill above the top of the trench, m (ft)

H_c = height of fill above top of conduit, m (ft)

H_p = height of conduit above level adjacent foundation, m (ft)

b_c = outside dimension of conduit, m (ft)

H_h = height of fill above horizontal diameter of conduit, m (ft)

(b) For low fills it may be desirable to use an effective height slightly less than H_h. The horizontal pressure for Condition II loading is determined using Equation 2-11.

$$p_e = \gamma H \tan^2 \left(45^\circ - \frac{\phi}{2} \right) + \left(\frac{H_f}{H_c + H_p} \right) \left[0.5\ \gamma H - \gamma H \tan^2 \left(45^\circ - \frac{\phi}{2} \right) \right] \tag{2-11}$$

where

H = variable height of fill above conduit, m (ft) (see definition, paragraph 2-4d(1)(a))

(c) For loading cases with submergence and water surcharge, the horizontal and vertical earth pressures should be similarly proportioned between the results obtained for Conditions I and III (trench and embankment conditions) with surcharge added to the hydrostatic pressure.

(3) Embankments (Condition III).

(a) Condition III applies to conduits and culverts that project above an embankment subgrade and to conduits and culverts in ditches that do not satisfy the requirements of Condition I or II. For this condition, the design should cover a range of possible loading conditions from the initial condition to the long-time condition by satisfying two extreme cases: Case 1, with $p_e/W_e = 0.33$ (W_e = 150 percent vertical projected weight of fill material, lateral earth pressure coefficient $k = 0.50$); and Case 2, $p_e/W_e = 1.00$ (W_e = 100 percent vertical projected weight of fill material, $k = 1.00$). The total vertical load in N/m (lbf/ft) for this condition should be computed as shown in Equations 2-12 and 2-13:

For Case 1, $W_e = 1.5\ \gamma b_c H_h$ (2-12)

For Case 2, $W_e = \gamma b_c H_h$ (2-13)

or the unit vertical load N/m^2 (psf), W_e, as given by Equations 2-14 and 2-15:

For Case 1, $W_e = 1.5\ \gamma\ H_h$ (2-14)

For Case 2, $W_e = \gamma\ H_h$ (2-15)

The horizontal loading N/m^2 (psf) should be taken as shown in Equations 2-16 and 2-17:

For Case 1, $p_e = 0.5\ \gamma\ H$ (2-16)

For Case 2, $p_e = \gamma\ H$ (2-17)

Normal allowable working stresses should apply for both Case 1 and Case 2.

(b) Where submergence and water surcharge are applicable, their effects must be considered as for Condition I. In such cases, the vertical load as computed by Equations 2-12 through 2-17, using the submerged weight of the material should be increased by the weight of the projected volume of water above the conduit including any surcharge water above the fill surface. When a clay blanket is applied to the face of the dam, the weight of water above the blanket must be included but the soil weight below the blanket and above the phreatic line (or the line of saturation where capillarity exists) is that for the natural drained condition. The horizontal unit pressure is found by adding full hydrostatic pressure to the value of p, obtained from Equation 2-16 or 2-17 using the submerged weight of the material.

2-5. Special Conditions

a. General. If conditions are encountered that warrant deviation from the loading criteria discussed above, justification for the change should be submitted with the analysis of design. However, the designer must first select the most economical method of installation. Where the rock surface occurs above the elevation of the bottom of the conduit, the designer should investigate the relative costs of excavating away from the conduit and backfilling between the conduit and the excavation line, allowing sufficient space between the conduit and the excavation line for operation of compaction rollers, and placing the conduit directly against rock as indicated for the following conditions.

b. Walls cast against rock. Where the conduit walls are placed directly against rock and the rock surface is at or above the top of the crown, the soil weight should be taken as 1.0 times the weight of material above, rather than 1.5, and the lateral pressure should be hydrostatic only, where applicable. Where the rock surface is at an intermediate level between crown and invert, use judgment to select a value between 1.0 and 1.5 to multiply by the weight of material above to obtain the correct soil design load. Lateral soil pressure should be applied only above the rock level and hydrostatic pressure as applicable over the full height of conduit. For either of these cases, the condition with no hydrostatic pressure should also be considered.

2-6. Methods of Analysis

Cast-in-place conduits can be designed using simplified elastic analysis or with finite element codes. Specialized finite element codes are available that feature nonlinear soil elements. These specialized codes provide the most accurate analysis. If these codes are not available, general finite element codes can be used, but they may need to be calibrated to the actual soil conditions. The finite element approach lends itself to parametric studies for rapid analysis of various foundation, bedding, and compaction conditions. Consult a geotechnical engineer for determination of soil spring constants to be used in the finite element model. Both concrete thickness and reinforcing steel area should be varied to obtain the best overall economy.

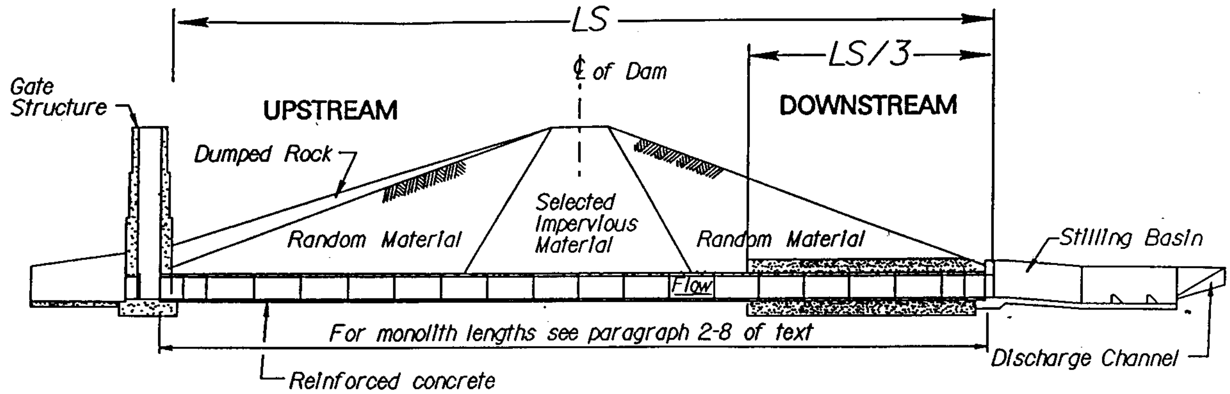
a. Finite element analysis. Finite element analysis is a useful method to design sections with unique shape for various field stresses. This method can be used to approximate the soil-structure interaction using spring foundations and friction between elements. These models calculate flexure and shear loads on the design section directly from soil-structure interaction relationships. The design of reinforcement for flexure and shear should be in accordance with EM 1110-2-2104. When the inside face steel is in tension, the area of steel needs to be limited to reduce the effects of radial tension. Therefore, limits on the amount of inside face steel that can be developed are necessary to prevent interior face concrete spalls or "slabbing failures." If more steel is required to develop the flexural capacity of the section, use radial ties. They should be designed in accordance with American Concrete Institute (ACI) 318 for shear reinforcement.

b. Curvilinear conduits and culverts (CURCON). This Computer-Aided Structural Engineering (CASE) program performs a structural analysis for conduit shapes including horseshoe, arch, modified oblong, and oblong sections with constant thickness, base fillets, or a square base. Loads that can be analyzed include groundwater and surcharge water in embankment backfills.

2-7. Reinforcement

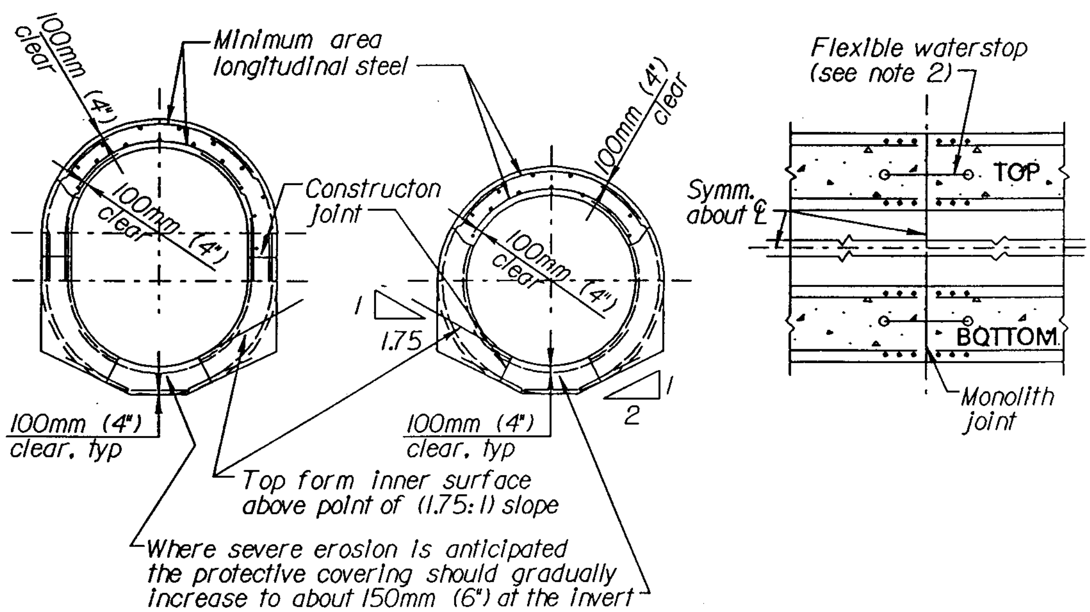
a. Minimum longitudinal. Longitudinal reinforcement should be placed in both faces of the conduit as shown in Figure 2-6. The minimum required area of reinforcement should not be less than 0.0028 times the gross area of concrete, half in each face, with a maximum of #30M at 300 mm (#9 at 12 in.) in each face. Generally, the same reinforcement will be in each face. Maximum spacing of bars should not exceed 450 mm (18 in).

b. Minimum transverse. Minimum transverse reinforcement should be placed in both inside and outside faces. Minimum required area of transverse steel, even when not carrying computed stresses, should not be less than 0.002 times the nominal area of concrete in each face, but not more than #25M at 300 mm (#8 at 12 in.) in each face, unless required to carry the computed stresses. Compression reinforcement in excess of this minimum should not be used.



RESERVOIR OUTLET WORKS–LONGITUDINAL SECTION THROUGH CONDUIT ON ROCK

Notes:
1. Conduit strength should vary roughly in accordance with height of overburden or other loading conditions so the overall structure will have essentially a constant safety factor throughout its length. Prefabricated conduit can usually be varied for strength class commercially available. For cast-in-place conduit both concrete thickness and reinforcing steel area should be varied to obtain the best overall economy.
2. The "Corps EM 1110-2-2102, Waterstops and other Joint Materials", illustrates various shapes of rubber and polyvinylchloride commercially available.



OBLONG SECTION

MODIFIED CIRCULAR SECTION

DETAIL SHOWING CONTRACTION JOINTS

Figure 2-6. Typical conduit details (large dams)

c. Minimum cover. Minimum concrete cover of reinforcement should not be less than 100 mm (4 in.).

2-8. Joints

a. Transverse monolith joints. Maximum contraction joint spacing should not exceed 6 m (20 ft) on earth foundations and 9 m (30 ft) on rock, as shown in Figure 2-6. When large settlements are expected, these maximum spacings should be reduced to allow for more movement in the joint. A geotechnical engineer should be consulted for soil settlements.

b. Longitudinal construction joints. The position of the longitudinal construction joints indicated in Figure 2-6 can be varied to suit the construction methods used. When circular and oblong conduits are used, the concrete in the invert section should be top-formed above the point where the tangent to the invert is steeper than 1 vertical on 1.75 horizontal.

2-9. Waterstops

Flexible-type waterstops should be used in all transverse contraction joints, as shown in Figure 2-6. Guidance on the selection of waterstop materials is given in EM 1110-2-2102. Where large differential movement is expected, a center-bulb-type waterstop and a joint separation of approximately 13 mm (1/2 in.) should be used. When the conduit rests on a rather firm foundation, a two-bulb or equivalent type waterstop should be used with a joint separation of approximately 6 mm (1/4 in.). For conduit on rock foundations with little expected deformation, the joint should be coated with two coats of mastic and an appropriate waterstop should be used.

2-10. Camber

When conduits are cast-in-place, large settlements are usually not a major concern. However, where considerable foundation settlements are likely to occur, camber should be employed to ensure positive drainage.